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EXPERIMENTAL AND NUMERICAL STUDIES OF THE SHEAR BEHAVIOUR OF LITESTEEL BEAMS

P. Keerthan[1] and M. Mahendran[2]

Abstract

This paper presents the details of experimental and numerical studies on the shear behaviour of a recently developed, cold-formed steel beam known as LiteSteel Beam (LSB). The LSB section is produced by a patented manufacturing process involving simultaneous cold-forming and electric resistance welding. It has a unique shape of a channel beam with two rectangular hollow flanges, made using a unique manufacturing process. To date, no research has been undertaken on the shear behaviour of LiteSteel beams with torsionally rigid, rectangular hollow flanges. In the present investigation, a series of numerical analyses based on three-dimensional finite element modelling and an experimental study were carried out to investigate the shear behaviour of 13 different LSB sections. It was found that the current design rules in cold-formed steel structures design codes are very conservative for the shear design of LiteSteel beams. Improvements to web shear buckling occurred due to the presence of rectangular hollow flanges while considerable post-buckling strength was also observed. Experimental and numerical analysis results are presented and compared with corresponding predictions from the current design codes in this paper.

Keywords: *Shear behaviour, LiteSteel Beams (LSB), Cold-formed steel structures, Slender web and hollow flanges.*

1.0 Introduction

In recent times cold-formed and thin-walled steel sections have been used extensively in residential, industrial and commercial buildings as primary load bearing members. The reasons for the popularity of cold-formed steel members include their wide range of applications, high strength to weight ratio, economy of transportation and handling, ease of fabrication and simple erection.

[1]PhD researcher, [2]Professor, School of Urban Development, Faculty of Built Environment & Engineering, Queensland University of Technology, Australia.

By taking advantage of the new material and manufacturing technologies and structurally efficient rectangular hollow flanges, Australian Tube Mills (ATM) has recently developed a new hollow flange channel section, known as the LiteSteel Beam (LSB) shown in Figure 1. Table 1 shows the nominal dimensions of LSB sections. In the large scale production of LSB sections, ATM uses the new dual electric welding and automated continuous roll-forming technologies for which it has worldwide patents. The innovative LSB sections have the beneficial characteristics of torsionally rigid closed rectangular flanges combined with economical fabrication processes from a single strip of high strength steel. They combine the stability of hot-rolled steel sections with the high strength to weight ratio of conventional cold-formed steel sections.

Flexural and shear capacities of LSBs must be known for LSBs to be used as flexural members. Flexural behaviour of LSBs has been investigated recently by Mahaarachchi and Mahendran (2005) by using experimental and numerical studies, and hence the moment capacities of LSBs are available. However, the shear behaviour of LSBs has not yet been investigated. Past research (Porter et al. 1975, Lee et al. 1995) has been restricted to plate girders and the shear buckling coefficient of the new mono-symmetric LSB sections has not been investigated. This paper presents the details of experimental and numerical studies of the shear behaviour of LSBs and the results.

Table 1: Nominal Dimensions of LSB

LSB Section	d (mm)	b_f (mm)	t (mm)	d_f (mm)
300x75x3.0	300	75	3	25
300x75x2.5	300	75	2.5	25
300x60x2.0	300	60	2	20
250x75x3.0	250	75	3	25
250x75x2.5	250	75	2.5	25
250x60x2.0	250	60	2	20
200x60x2.5	200	60	2.5	20
200x60x2.0	200	60	2	20
200x45x1.6	200	45	1.6	15
150x45x2.0	150	45	2	15
150x45x1.6	150	45	1.6	15
125x45x2.0	125	45	2	15
125x45x1.6	125	45	1.6	15

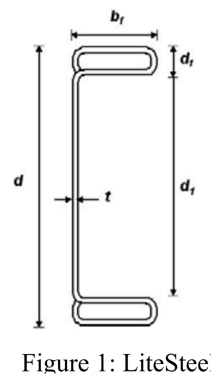


Figure 1: LiteSteel Beam

2.0 Experimental Study

Shear behaviour of LSBs was investigated using a series of pure shear tests of simply supported LiteSteel beams subjected to a mid-span load (see Figure 2). In order to simulate a pure shear condition, relatively short test beams of span based on aspect ratio (shear span a/ clear web height d_1) of 1 & 1.5 were selected. Two LSB sections were bolted back to back using three T-shaped stiffeners located at the end supports and the loading point in order to eliminate any torsional loading of test beams.

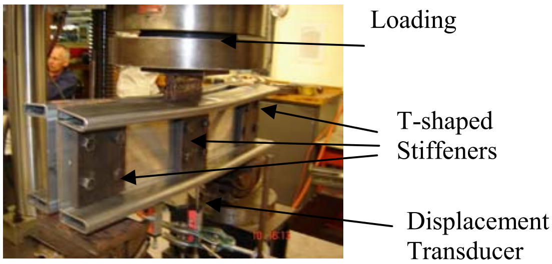


Figure 2: Experimental Set-up

The stiffeners were used to avoid eccentric loading and web crippling. A 20 mm gap (see Figure 2) was included between the sections to allow the test beams to behave independently while remaining together to resist torsional effects. Figure 2 shows the experimental set-up used in this research.

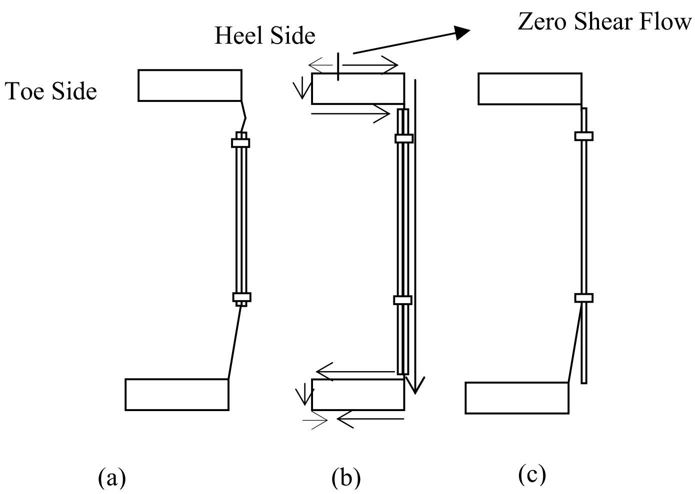


Figure 3: Effects of Web Side Plate (WSP)

Table 2: Experimental Results

Test No	LSB Section	Aspect Ratio	WSP Details	s/d_l %	Ult. Load (kN)	Failure Mode
1	125x45x2.0	1.55	Both sides: 90x75	95	56.94	Shear Yielding
2	150x45x1.6	1.54	Both sides: 90x75	75	41.67	Inelastic Shear Buckling
3	150x45x1.6	1.00	Both sides: 90x75	75	43.50	Shear Yielding
4	150x45x2.0	1.00	Both sides: 90x75	75	61.22	Shear Yielding
5	150x45x2.0	1.54	Both sides: 90x75	75	53.84	Shear Yielding
6	200x45x1.6	1.50	Both sides:140x75	82	45.50	Elastic Shear Buckling
7	200x45x1.6	1.50	Both sides:156x75	92	54.19	Elastic Shear Buckling
8	250x60x2.0	1.50	One side: 206x75	98	61.12	Inelastic Shear Buckling
9	250x60x2.0	1.50	Both sides:206x75	98	>75	Inelastic Shear Buckling
10	200x60x2.0	1.50	Both sides:156x75	98	73.98	Inelastic Shear Buckling
11	300x60x2.0	1.50	Both sides:246x75	95	>75	Elastic Shear Buckling

Note: WSP sizes are given as height (s) x width; d_l= Depth of flat portion of web measured along the plane of the web.

Table 2 shows the details of the test specimens used and the results. In Tests 2 to 6, a tendency of the LSB flanges to displace laterally was observed (see Figures 3 (a) and 4). At the connection, the top flange of the LSB tended to displace laterally towards the heel side of the flange while the bottom flange would displace towards the opposite side (the toe side). This occurred when the full depth of web element of LSB was not supported by the web side plate (WSP), ie. the WSP height (s) was less than then web height (d_l). This led to reduced restraint to the lateral movement of flanges. When full lateral support was provided to the LSB top and bottom flanges at the connections by using WSPs with full web height as shown in Figures 3 (b) and 5, the LSB top and bottom flanges were effectively prevented from lateral displacement at the connections. The results from Tests 6 and 7 show that the shear capacity of LSB increases with increasing height of web side plate (WSP).

In Test 8, one WSP was used to investigate its effect on the shear capacity of LSB (see Figures 3(c) and 6) where LSB top flange was effectively prevented from lateral displacement at the connections by outside (Heel side) WSP while the bottom flange would displace towards the opposite side (Toe side). This occurred because the web element was not fully supported inside by the WSP (Toe side). When the results of Test 8 (WSP on one side only) and Test 9 (WSP on both sides) are compared, there is more than 19% capacity reduction due to the lateral movement of the bottom flange. To prevent the lateral movement of bottom flange, bolts should be located near the bottom flange. More shear tests are being undertaken at present using WSPs on both sides with a height equal to that of LSB web element (d_1).

Figure 4: Web with Two Partial WSP for 200x45x1.6 LSB

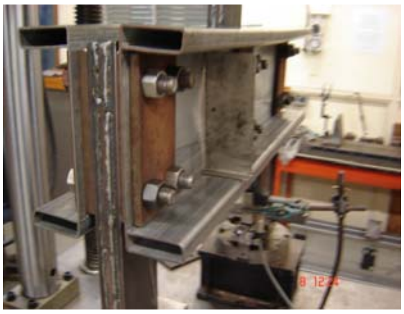


Figure 5: Web with Two Full WSP for 200x45x1.6 LSB

Figure 6: Web with by One Full WSP for 200x45x1.6 LSB

3. Shear Yielding Behaviour of Beam Web Panels

3.1 General

A stocky web (small depth to thickness ratio) is subjected to shear yielding. The section yields, but does not buckle, as the web is compact. The stocky web section will yield in shear at an average stress of $f_y / \sqrt{3}$ as given by the von Mises yield criterion (Hancock, 1998). The nominal shear yielding capacity of the section is therefore given by Equation 1**.** Figure 7 shows the shear yielding of LiteSteel beam. The accuracy of this equation in predicting the shear capacity of LSBs will be discussed in Section 5 by comparing with experimental results.

$$V_v = 0.64 f_y d_1 t_w \qquad \text{for} \quad \frac{d_1}{t_w} \leq \sqrt{\frac{Ek_v}{f_y}} \tag{1}$$

where d_1= Depth of flat portion of web measured along the plane of the web, t_w = Thickness of the web f_y , E =Yield stress used in design and Modulus of elasticity of steel; k_v = Shear buckling coefficient.

Figure 7: Shear Yielding Failure
(125x45x2 LSB)

4. Shear Buckling Behaviour of Beam Web Panels

4.1 General

For a web element with a large depth to thickness ratio, its shear capacity is governed by elastic shear buckling. The elastic critical shear buckling stress can be computed by Equation 2 (Hancock, 2005). Equation 3 gives the shear capacity (V_v) of conventional cold-formed steel beams in the case of elastic shear buckling.

$$\tau_{cr} = \frac{k_v \pi^2 E}{12(1-\nu^2)} \left(\frac{t_w}{d_1} \right)^2 \tag{2}$$

where k_v = Shear buckling coefficient (5.34) and other symbols have been defined in Eq. (1).

$$V_v = \frac{0.905\ E k_v t_w^3}{d_1} \quad \text{for} \quad 1.415\sqrt{\frac{Ek_v}{f_y}} < \frac{d_1}{t_w} \tag{3}$$

In the region where shear buckling and yielding interact, the failure stress is given by the geometric mean of the buckling stress and 0.8 times the yield stress in shear (Hancock, 1998). In the case of inelastic shear buckling the resulting equation for the nominal shear capacity (V_v) is given by Equation 4.

$$V_{vw} = 0.64 t_w{}^2 \sqrt{Ek_v f_y} \quad \text{for} \quad \sqrt{\frac{Ek_v}{f_y}} < \frac{d_1}{t_w} \le 1.415\sqrt{\frac{Ek_v}{f_y}} \tag{4}$$

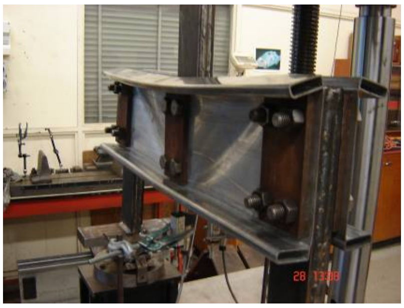


Figure 8: Elastic Shear Buckling 200x45x1.6 LSB

Figure 9: Inelastic Shear Buckling 200x60x2 LSB

Figure 8 shows the elastic shear buckling of LSB while Figure 9 shows the inelastic shear buckling of LSB. The boundary condition at the juncture of the web and flange elements is somewhere between simple and fixed condition as recognized from early days. Such conservative assumption was made mainly due to the inability to evaluate it in a rational manner. For example, Basler (1961) and Porter et al. (1975) assumed that the web panel was simply supported at the juncture while Chern and Ostapenko (1969) obtained the ultimate strength by assuming that the juncture behaved like a fixed support.

The boundary condition at the flange-web juncture in practical designs is much closer to fixity for the plate girders (Lee et al. 1995). Therefore the assumption that the web panel is simply supported at the juncture sometimes leads to a considerable underestimation of the ultimate shear strength because of the underestimation of the elastic shear buckling strength of plate girders. Based on a numerical study, Lee et al. (1995) proposed simple equations to determine the shear buckling coefficients (k_v) of plate girder web panels. A similar approach was used in this investigation for LSBs.

4.2 Elastic Buckling Analysis

In order to obtain the shear-buckling coefficient of LSBs, finite element analyses were carried out using ABAQUS based on the ideal model of LSB with aspect ratios (shear span a/web height d_1) of 1 (see Figure 10). The ideal models included the nominal web and flange yield stresses of 380 and 450 MPa, respectively. These yield stresses are the minimum specified values for the range of LSB sections. Finite element model was to provide "idealized" simply supported boundary conditions. Element widths of 5 mm x 5 mm were selected as the suitable mesh size through the entire cross-section for LSB sections. The shear flow pattern loading was applied to prevent the twisting effect. These shear flow pattern loadings are calculated by using the principal shear flow equation. The boundary conditions of finite element models are given in Table 3. Figure 11 shows the shear buckling mode of LiteSteel beam.

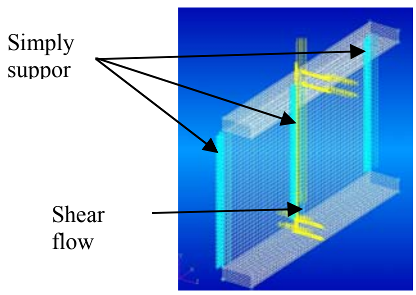


Figure 10: Ideal Finite Element Model (200x45x1.6 LSB)

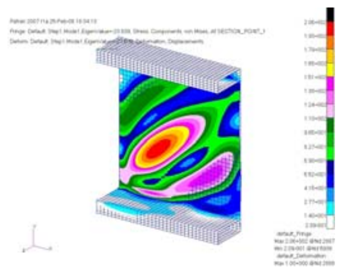
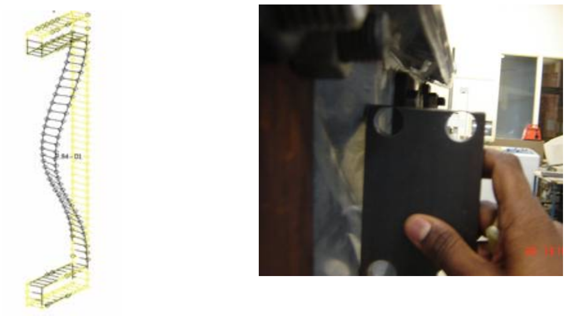


Figure 11: Shear Buckling Mode (200x45x1.6 LSB)

Table 3: Boundary Conditions Used in the Finite Element Model

Edges	u	v	w	θ_x	θ_y	θ_z
Left and Right	0	1	1	1	0	0
Middle	1	0	1	1	0	0

Note: u, v and w are translations and θ_x, θ_y and θ_z are rotations in the x, y and z directions, respectively. 0 denotes free and 1 denotes restraint.



(a) 200x45x1.6 LSB
Finite Element Model

(b) 200x45x1.6 LSB
Experimental Model

Figure 12: Shear Buckling Deformation of LSB

Figure 12 (a) shows the deformed cross sections of the buckled LiteSteel beam. Deformed cross-section of web panels resemble the buckling mode shape of Eulerian column fixed at both ends. This observation implies that the boundary condition at the flange-web juncture of LSBs is very close to a fixed support condition. This observation was confirmed by the shear tests as shown in Figure 12 (b).

Table 4 compares the shear buckling coefficients ($k_{\mathbf{LSB}}$) determined from the eigenvalue analysis and Equation 2 for the aspect ratio of 1. Shear buckling coefficients of plate with simple-simple and simple-fixed boundaries, k_{ss} and $k_{sf,}$ were determined by using Equations 5 and 6, respectively. Table 4 indicates that k_{LSB} is very close to k_{sf}. Therefore the realistic support condition of LSB at the web-flange juncture is closer to a fixed condition.

$$k_{ss} = 5.34 + \frac{4}{\left(\frac{a}{d_1}\right)^2} \qquad \text{for} \qquad \frac{a}{d_1} \geq 1 \tag{5}$$

$$k_{sf} = 8.98 + \frac{5.61}{\left(\frac{a}{d_1}\right)^2} - \frac{1.99}{\left(\frac{a}{d_1}\right)^3} \qquad \text{for} \qquad \frac{a}{d_1} \geq 1 \tag{6}$$

where a = Shear span of web panel and other symbols have been defined in before.

Table 4: Comparison of Shear Buckling Coefficients of LiteSteel Beams (Aspect Ratio =1)

LSB Section	k_{ss}	k_{sf}	k_{LSB}
125x45x1.6	9.34	12.6	12.58
125x45x2.0	9.34	12.6	12.59
150x45x1.6	9.34	12.6	12.57
150x45x2.0	9.34	12.6	12.58
200x45x1.6	9.34	12.6	12.19
200x60x2.0	9.34	12.6	12.57
200x60x2.5	9.34	12.6	12.58
250x60x2.0	9.34	12.6	12. 45
250x75x2.5	9.34	12.6	12.58
250x75x3.0	9.34	12.6	12.59
300x60x2.0	9.34	12.6	12.41
300x75x2.5	9.34	12.6	12.43
300x75x3.0	9.34	12.6	12.45

4.3 Shear Buckling Coefficient

Based on the results from the finite element elastic buckling analyses the following simple equation (Equation 7) was found to determine the shear buckling coefficients of LiteSteel beams. Here the minimum shear buckling coefficient of LSB (12.19 from Table 4) was taken to propose the formula for aspect ratio $\frac{a}{d_1} \geq 1$. Since longer span LiteSteel beams are being used in practical applications, the aspect ratio greater than or equal to one was considered. The values of k_{ss} and k_{sf} for a given aspect ratio were determined from Equations 5 and 6, respectively.

$$k_{LSB} = k_{ss} + 0.87(k_{sf} - k_{ss}) \qquad \text{for} \qquad \frac{a}{d_1} \geq 1 \qquad (7)$$

This equation is similar to that proposed by Lee at al. (1995) for the shear buckling coefficient of plate girders. Proposed shear buckling coefficient equation for LiteSteel beam (Equation 7) shows that the boundary condition at flange-web juncture of LSBs is equivalent to 87% fixed condition. It is noted that the boundary condition at flange-web juncture of LSBs is almost the same as that for plate girders as Lee et al. (1995) obtained 82% fixity.

4.4 New Proposed Formula for the Shear Strength of LiteSteel Beams

New design shear strength formulae were proposed for LSBs based on the design equations given in AS/NZS 4600. The increased shear buckling coefficient for LSB as given by Equation 7 is included here to allow for the additional fixity in the web-flange juncture. However, post-buckling strength was not included. Equations 8 to 10 present the relevant design equations.

$$\tau = 0.64 f_y \qquad \text{for} \qquad \frac{d_1}{t_w} \leq \sqrt{\frac{Ek_{LSB}}{f_y}} \quad \text{(Shear yielding)} \tag{8}$$

$$\tau = \frac{0.64\sqrt{(Ek_{LSB} f_y)}}{\left[\dfrac{d_1}{t_w}\right]} \qquad \text{for} \quad \sqrt{\frac{Ek_{LSB}}{f_y}} < \frac{d_1}{t_w} < 1.415\sqrt{\frac{Ek_{LSB}}{f_y}} \quad \text{(Inelastic shear buckling)} \tag{9}$$

$$\tau = \frac{0.905 Ek_{LSB}}{\left(\dfrac{d_1}{t_w}\right)^2} \qquad \text{for} \quad \frac{d_1}{t_w} \geq 1.415\sqrt{\frac{Ek_{LSB}}{f_y}} \quad \text{(Elastic shear buckling)} \tag{10}$$

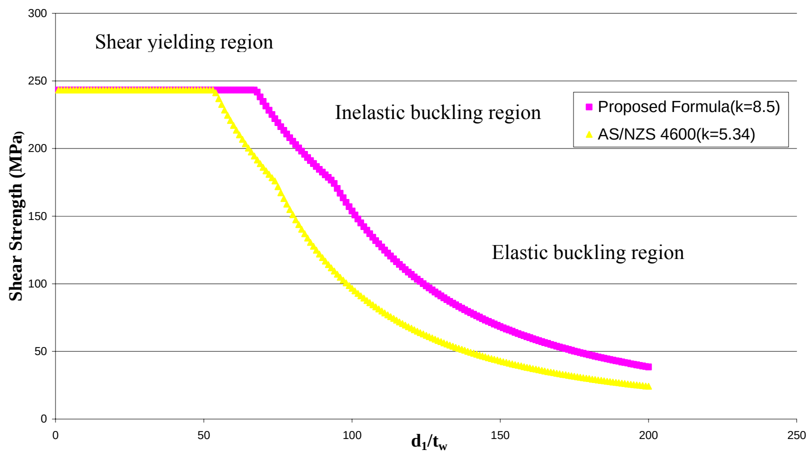


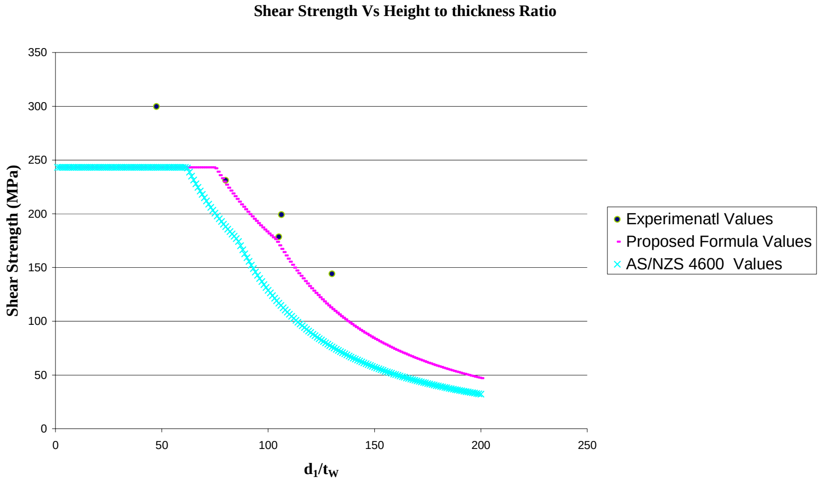
Figure 13: Shear Strength of LSB for Infinity Aspect Ratio versus Web Height to Thickness Ratio.

Longer span LiteSteel beams without transverse stiffeners are commonly used in practical applications. In order to simulate this practical application, an aspect ratio of infinity was considered. Figure 13 shows the new design curves based on the proposed equations (8 to 10) for the aspect ratio of infinity in comparison to the original AS/NZS 4600 design equations. It shows that the shear capacities predicted by the current design rules in AS/NZS 4600 are conservative because AS/NZS 4600 (SA, 2005) assumes that the web panel is simply supported at the juncture between the flange and web elements (uses a k_v of 5.34). However in this study it was found that the realistic support condition at the web-flange juncture of LSB is closer to a fixed support condition that gives a k_v of 8.5. Therefore the assumption considered by Clause 3.3.4 of AS/NZS 4600 may result in an overly conservative shear design for LSBs.

5.0 Comparison of Proposed Design Formulae and Experimental Capacities

Proposed shear design formulae are valid when the WSPs are used to the full height of the web element at the supports (no lateral movements of top and bottom flanges). In Tests 1, 7, 9, 10 and 11, the WSP height was more than 90% of LSB web element height (see Table 2). Therefore these experimental results can be compared with the proposed design formulae. New shear strength formulae predictions are compared with experimental strengths in Table 5. Figure 14 shows the new design curves based on the proposed equations (8 to 10) for the aspect ratio of 1.5, and compares them with the experimental capacities and AS/NZS 4600 design equations. It shows that the shear capacities predicted by the current design rules in AS/NZS 4600 are very conservative while the proposed design formulae are also conservative as the potential post-buckling strength has not been included.

Table 5: Comparison of Ultimate Shear Strengths from Experiments and Proposed and Current Design Formulae



LSB Section	Aspect Ratio	Ultimate Shear Strength (MPa)			Failure Mode
		Experimental Results	Proposed Formula	AS/NZS 4600	
125x45x2.0	1.55	56.94	49.64	49.64	Shear yielding
200x45x1.6	1.50	54.19	46.00	31.47	Elastic Shear Buckling
200x60x2.0	1.50	73.98	72.50	59.97	Inelastic Shear Buckling
250x60x2.0	1.50	>75	72.50	59.97	Inelastic Shear Buckling
300x60x2.0	1.50	>75	57.86	39.6	Elastic Shear Buckling

Figure 14: Shear Strength of LSB versus Web Height to Thickness Ratio (d_1/t_w). Aspect Ratio =1.5

Plates with a large width to thickness ratio when subjected to direct compression or shear undergo elastic buckling at a critical stress value. Analytical studies show that thin plates do not collapse when buckling stress is reached, but has considerable post-buckling strength. This has been experimentally verified for plates under axial compression and appropriate strength formulae have also been developed and included in various codes. However, this is not the case for shear loading. Presumably because of lack of experimental evidence on shear capacity of plates without stiffeners, design codes do not include the post-buckling strength in shear, and the design shear stress in webs is therefore limited by the elastic buckling capacity (Suter and Humar, 1986). This research has shown that significant reserve strength beyond elastic buckling is present and that post-buckling shear strength in LSB can be included in their design (Fig.14). Further research is currently under way using both experimental and numerical studies.

6. Conclusion

This paper has presented the details of an investigation into the shear behaviour of an innovative cold-formed hollow flange channel section known as LiteSteel beams. Experimental studies were performed to investigate the shear behaviour of LSBs while advanced finite element analyses were used to investigate their elastic shear buckling behaviour.

It was found that AS/NZS 4600 design equations can be used conservatively for LSBs undergoing shear yielding. The current shear capacity design rules for LSBs are based on Clause 3.3.4 of AS/NZS 4600 where the web panel is considered simply supported at the juncture between flange and web elements. However, this study has shown that the realistic support condition at the web-flange juncture of LSB is closer to a fixed support condition and therefore the assumption considered by Clause 3.3.4 of AS/NZS 4600 may result in an overly conservative shear design for LSBs. It was found that significant reserve strength beyond elastic buckling is present and that post-buckling shear strength can be included in design. Appropriate improvements have been proposed for the shear strength of LSBs based on AS/NZS 4600 design equations.

7. Acknowledgements

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